



BEARING CAPACITY OF FOUNDATIONS RESTING ON A TRENCH OF LOCAL RECLAIMED ASPHALT PAVEMENT MATERIAL

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ABSTRACT

One of the methods used to improve the bearing capacity of footings constructed over soft clays is by replacement of the soft soil by making a trench filled with suitable material. Typical dimensions for an over excavation width of the bottom of the excavation typically varies from one to three times the width of the foundation and the depth below the bearing level is generally about $\frac{1}{2}$ to $1\frac{1}{2}$ times the foundation width (0.5B to 1.5B). If a good bearing stratum (medium dense sand, dense sand, gravel, or bedrock) exists close to the bearing level, the excavation is usually taken to the top of the bearing stratum or a shallow depth into it.

The present study is intended to make use of the reclaimed asphalt pavement (RAP) as a replacement material underneath a footing resting on soft clay. A trench of replaced soil will be extended to different depths to investigate the suitable dimensions of the trench.

It was concluded that there is increase in the bearing capacity ratio when a trench of RAP material is constructed underneath the footing. This increase depends on the width and depth of the trench. The bearing capacity of the square footing increased by about 42% when a trench of width B and 0.2 B depth is constructed underneath the footing. This increase becomes 61% when the trench width is made ($W = 1.6 B$). On the other hand, the bearing capacity of the strip footing increased by about 50% when a trench of width B and 0.33 B depth is constructed underneath the footing. This increase becomes 65% when the trench width is made ($W = 2 B$).

INTRODUCTION

One of the primary criteria used to select a site for development is the suitability of the ground for supporting the structure to be built. In most urban areas, the best sites were developed first, and, as urbanization continues, when a previously undeveloped site is purchased, the engineering properties of the existing near-surface materials are often such that the structure cannot be supported by shallow foundations. The traditional solution for these situations is to support the structures on deep foundations—typically piles or drilled piers—where a small portion of the load is transmitted to the poorer near-surface materials and a large portion of the load is transmitted to better bearing materials deeper within the ground. However, even with the development and use of more economical types of deep foundations such as auger-cast piles, the demand has increased for more economical solutions to the unsuitability of near-surface soils for shallow foundations.

To meet this demand, numerous soil stabilization and improvement techniques (also commonly called ground modification techniques) have been developed within the past 30 years or so. These techniques involve modifying the engineering properties and behavior of the near-surface soils at a site so that shallow foundations can be used where they previously could not or in some instances so that more economical shallow foundations can be used (Lawton, 2004).

OVEREXCAVATION / REPLACEMENT OF SOIL

The technique of overexcavation/replacement is one of the oldest, most intuitive, and simplest methods for modifying bearing materials to increase support for shallow foundations. The method consists of excavating poor or inadequate bearing material and replacing it with a stiffer and stronger material (Figure 1). As long as the in-place replacement material is stiffer and stronger than the excavated bearing soil, the settlement that the foundation undergoes when loaded is reduced and the factor of safety against ultimate bearing capacity failure is increased. The greater the stiffness and strength of the replacement material, the greater the reduction in settlement and increase in ultimate bearing capacity (Lawton, 2004).

Overexcavation/replacement is most commonly used when the bearing soils are very weak and highly compressible. The replacement material can be the excavated material that has been modified in some way, or it can be borrow material (obtained from another location on or off the site). The replacement material is usually sand, gravel, or a sand-gravel mixture, especially in situations where the ground-water table is high or when it is

desirable to have a free-draining bearing material. To obtain the stiffest and strongest material possible, the replaced sand or gravel is usually compacted in lifts during the replacement process. If good-quality sands and gravels are not readily or economically available, the excavated soils can be chemically stabilized and used as the replacement material.

However, chemically stabilized soils are generally not free-draining (they have relatively low permeabilities), so their use as a replacement material may change the local or regional ground-water seepage and precipitation infiltration patterns, which may be an environmental consideration in some instances. It is also possible to use the excavated soil as the replacement material without chemically modifying it, although this is seldom done. If the inadequate bearing soil is cohesionless, several techniques are available for densifying in situ soil that are more economical than removing, drying or wetting, replacing, and compacting it. Cohesive soils can be excavated, dried, and compacted at an appropriate water content to produce a material that is initially stiff and strong, but these soils may be susceptible to wetting-induced volume changes and reductions in stiffness and strength from wetting.

The excavation is deeper and often wider than is needed to place the foundation, hence the term overexcavation. Typical dimensions for an overexcavation are shown in Figure 1. The width of the bottom of the excavation typically varies from one to three times the width of the foundation (B to $3B$ in Figure 1), and the depth below the bearing level is generally about $\frac{1}{2}$ to $1\frac{1}{2}$ times the foundation width ($0.5B$ to $1.5B$). If a good bearing stratum (medium dense sand, dense sand, gravel, or bedrock) exists close to the bearing level, the excavation is usually taken to the top of the bearing stratum or a shallow depth into it (Lawton, 2004).

The replacement material is usually compacted so that the replaced zone is as stiff and strong as possible. A variety of compaction procedures can be used. If the excavation is narrow and shallow, hand-operated compaction equipment appropriate for confined areas is used, including rammers, tampers, vibrating plates, and small rollers. For deep, narrow excavations, backhoes or hydraulic excavators with special compaction attachments can be used. Other techniques include pounding the material with the bucket of a backhoe and dropping a weight from a crane. For wide excavations, full-size compaction rollers are normally used (Lawton, 2004).

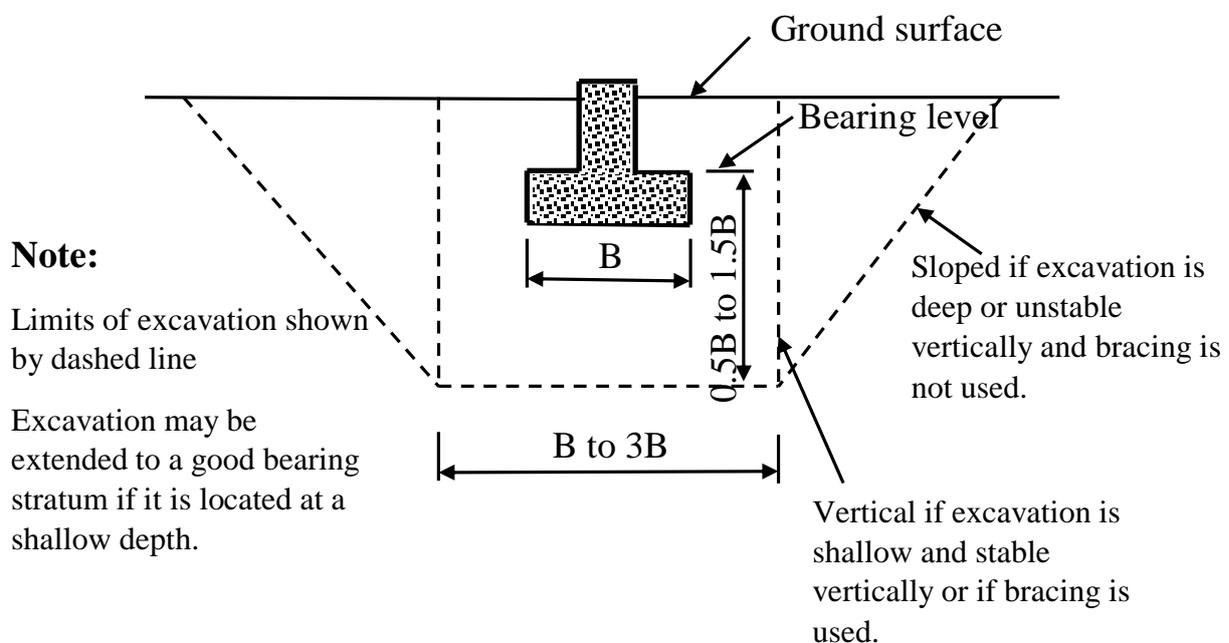


Fig. 1: Typical dimensions of excavation for overexcavation/replacement (Lawton, 2004).

A theoretical solution for this case, where the foundation bears on a long (continuous) granular trench within a soft saturated clay matrix immediately after loading (the clay is undrained), has been given by Madhav and Vitkar

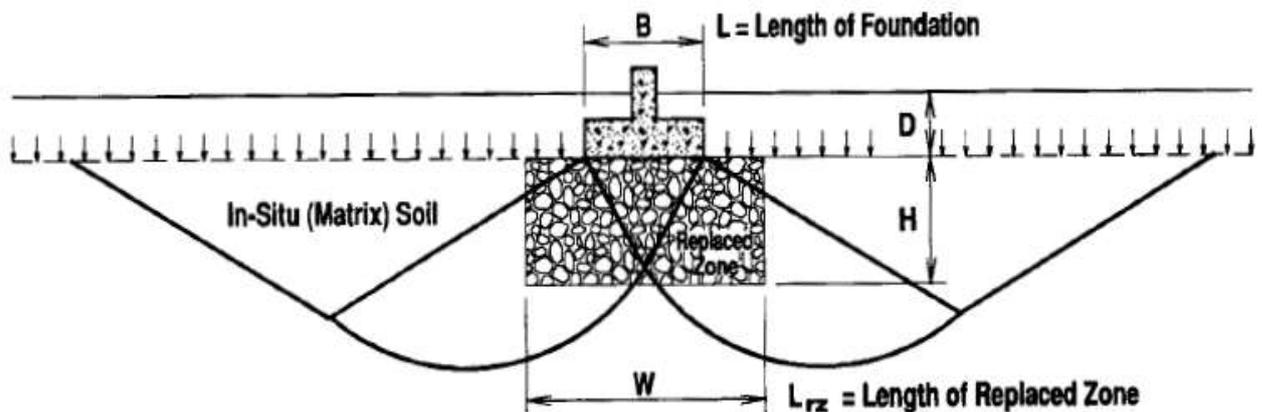


(1978). Their solution is presented in the form of charts from which modified bearing capacity factors (N_{ct} , N_{qt} , and $N_{\sigma t}$) can be obtained.

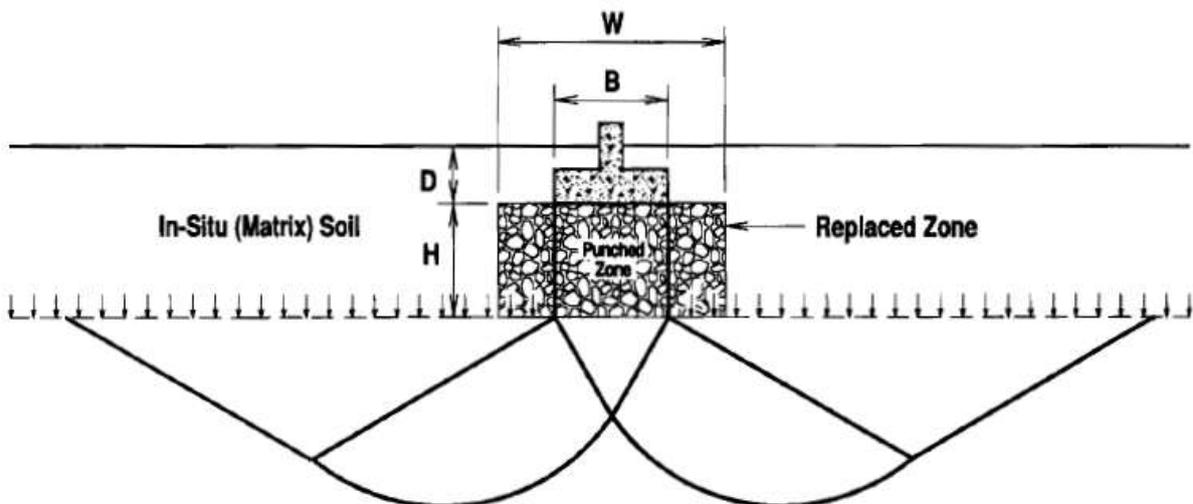
Hamed et al. (1986) conducted laboratory model tests to determine the variation in the ultimate bearing capacity q_{ult} for a continuous foundation bearing on a granular trench (width of trench $W = B$) constructed within a soft clay matrix. By comparing their experimental values of q_{ult} with values predicted from a derived equation, Hamed and colleagues concluded that Madhav and Vitkar's theory overpredicts q_{ult} . In addition, they found that the minimum height of the granular trench necessary to obtain the maximum value of q_{ult} is about $3B$.

Three possible failure methods are shown in Figure 2 for a foundation bearing on a strong replaced zone of finite width and depth. It is possible (but unlikely) that the most critical failure surface is a general bearing failure surface that develops through the replaced zone and into the in situ soil (Figure 2a; a theoretical solution for this case where the foundation bears on a long (continuous) granular trench within a soft saturated clay matrix immediately after loading (the clay is undrained) has been given by Madhav and Vitkar (1978).

If the replaced zone punches through the matrix soil [Figure 2c], punching resistance will develop within the in situ soil adjacent to the vertical perimeter surface of the replaced zone.



a) *general shear failure through replaced zone and in situ soil.*



b) *punching failure through replaced zone.*

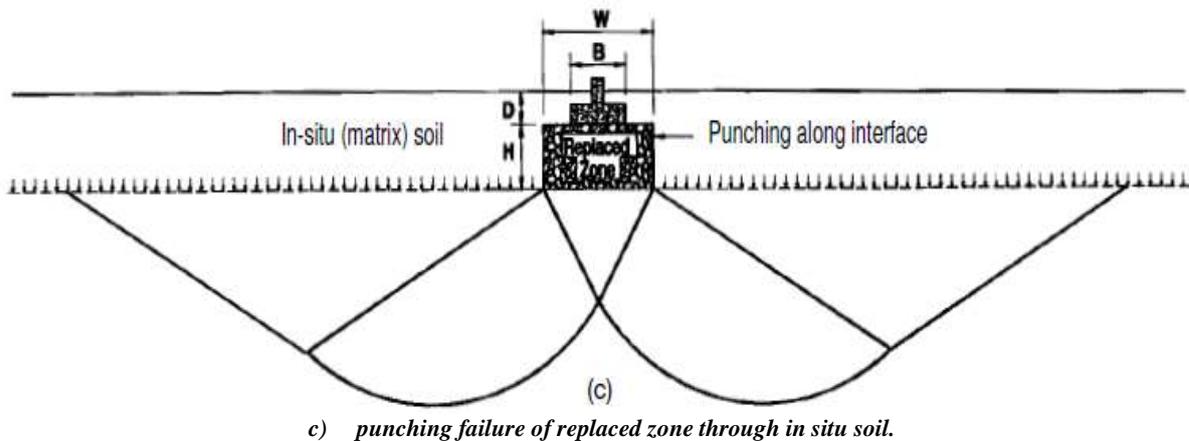


Fig. 2: Potential bearing shear failure mechanisms for foundation supported by overexcavated and replaced zone (after Madhav and Vitkar, 1978).

Settlement

When properly designed and implemented, overexcavation and replacement results in reduced settlement owing to one or both of the following factors: (a) The replaced zone is stiffer so it settles less than the replaced in situ soil would have, and (b) the vertical stresses induced in the in situ soil beneath the replaced zone may be less than without the replaced zone, so the settlement of this underlying soil may also be less. As will be discussed subsequently, the vertical stresses induced in the underlying in situ soil may be greater with a replaced zone than without it, so care must be exercised when using overexcavation/replacement where saturated fine-grained soils are within the zone of influence for settlement (Lawton, 2004).

Fattah et al. (2010) provided a finite element procedure to model soft cohesive soil, granular trench soil, and the reinforcement material by using a computer program called (SIGMA/W). The behavior of both cohesive and granular soils was simulated by nonlinear-elastic soil model (hyperbolic model), while the linear-elastic model was used to simulate the reinforcement material. The angle of friction of trench soil, modulus of elasticity of reinforcement material, depth, width and shape of the granular trench, locations, and number of the reinforcement layers were varied. The sloped granular trench was analyzed in two cases; lined and unlined conditions. The results showed that the use of granular trench beneath a foundation will increase the bearing capacity and reduce the settlement. Moreover, using of polymers as a reinforcement material has a significant effect on both bearing capacity and settlement. For both reinforced and unreinforced granular trenches, the depth ratio has an important effect on the settlement ratio, which decreases with the increase of depth ratio. The best practical value for the depth ratio was found to be equal to 2. Making a trench with a width (W) larger than the foundation width (B) also decreases the settlement, and the best effect occurs when the width ratio (W/B) equals to 0.65.

Influence of providing a Granular Trench (GT) below strip footings on loose sand deposits was explored by Unnikrishnan et al. (2011). The additional benefit of encapsulating such a granular trench with a geosynthetic was also studied. Such a system is christened as Encapsulated Granular Trench (EGT). Load tests were conducted on laboratory model strip footings resting on granular trench and encapsulated granular trench. EGT supported strip footing was found to perform better than the footing on GT. The geosynthetic helps to redistribute the stresses within the granular trench and undergo self-straining due to loading. In addition to this, the geosynthetic will also perform as a separator. Nonlinear finite element analyses were conducted to understand the internal mechanisms leading to the observed behavior.

The reinforcement of the problematic soils with granular fill layers is one of the soil improvement techniques that are widely used. Problematic soil behavior can be improved by totally or partially replacing the inadequate soils with layers of compacted granular fill. Ornek et al. (2012a) presented the use of artificial neural networks (ANNs), and the multi-linear regression model (MLR) to predict the bearing capacity of circular shallow footings supported by layers of compacted granular fill over natural clay soil. The data used in running the network models have been obtained from an extensive series of field tests, including large-scale footing diameters. The field tests were performed using seven different footing diameters, up to 0.90 m, and three different granular fill layer thicknesses.



The results indicated that the use of granular fill layers over natural clay soil has a considerable effect on the bearing capacity characteristics and that the ANN model serves as a simple and reliable tool for predicting the bearing capacity of circular footings in stabilized natural clay soil.

Numerical predictions of the scale effect for circular footings supported by partially replaced, compacted, layers on natural clay deposits were presented by Ornek et al. (2012b). The scale effect phenomenon was analyzed according to the footing sizes. Numerical analyses were carried out using an axisymmetric, two-dimensional, finite-element program. Before conducting the analysis, the validity of the constitutive model was validated using field tests performed by authors with seven different footing diameters up to 0.90 m and with three different partial replacement thicknesses. It was shown that the behavior of the circular footings on natural clay soil and the partial replacement system can be reasonably well represented by the Mohr Coulomb model. The Mohr-Coulomb model parameters were derived from the results of conventional laboratory and field tests. After achieving a good consistency between the results of the test and the numerical analysis, the numerical analyses were continued by increasing the footing diameter up to 25 m, considering the partial replacement thickness up to two times the footing diameter. The results of this parametric study showed that the stabilization had a considerable effect on the bearing capacity of the circular footings and for a given value of H/D the magnitude of the ultimate bearing capacity increases in a nonlinear manner with the footing diameter. The Bearing

Capacity Ratio (BCR) was defined to evaluate the improved performance of the reinforced system. It was found, based on numerical and field-test results that the BCR of the partially replaced, natural clay deposits increased with an increase in the footing diameter and there was no significant scale effect of the circular footing resting on natural clay deposits.

Gueguin et al. (2015) addressed the geotechnical engineering problem of evaluating the ultimate bearing capacity of a strip foundation resting upon a reinforced soil, by means of the yield design homogenization approach. The analysis was notably focused on the determination of the macroscopic strength criterion of such reinforced soils, where both constituents are purely cohesive, which can be conveniently expressed through the notion of anisotropic cohesion. A comprehensive comparison was made between the classical configuration of reinforcing columns and the more original one of orthogonal reinforcing trenches. Among the most outstanding results of the analysis is the conclusion that the cross trench configuration is notably more efficient in terms of load bearing capacity than the reinforcement by columns, notably when significantly inclined loading is concerned.

Response of strip footings in loose sand deposits supported on granular trenches was explored by Unnikrishnan et al. (2010). Non-linear finite element simulations were carried out to investigate the influence of granular trench on the load settlement behavior of strip footings. Load test was carried out on model strip footings with and without granular trench support to validate the finite element formulation. Sea sand placed with low relative density was used as the bulk material. Well graded river sand was used as the replacement soil in the granular trench. The soils were characterized through laboratory tests. Influence of parameters such as size and shape of the trench were investigated through non-linear finite element simulations. The investigations revealed that the provision of granular trenches beneath footings improve the response of strip footings supported on them.

El-Sawwaf and Nazir (2012) presented an experimental study of the behavior of an eccentrically loaded model ring footing resting on a compacted replaced layer of sand that overlies an extended layer of loose sand. Particular emphasis was placed on the potential benefits of reinforcing the replaced sand layer with geogrid reinforcement. Load configuration was designed to simulate ring footings under vertical loads and overturning moment caused by lateral loads. Several configurations of geogrid layers, number, and stiffness were used to study ring footings with different inner-to-outer-diameter ratios and load eccentricities. The effect of the depth and the relative density of the replaced sand layer was also investigated. Test results indicated that the behavior of an eccentrically loaded ring footing significantly improves with an increase in the depth and the relative density of the replaced compacted sand layer. However, the inclusion of soil reinforcement not only leads to a significant reduction in the depth of the replaced sand layer but also causes a considerable increase in the bearing capacities of the eccentrically loaded rings, leading to the cost-effective design of the footings.

RECLAIMED ASPHALT PAVEMENT MATERIAL

The technique of Reclaimed Asphalt Pavement (RAP) for road construction is widely used so that RAP has been called the most recycled material in the world. RAP is most commonly used as an aggregate and asphalt binder



substitute in recycled asphalt paving, but it is also used as a granular base or sub-base, stabilized base aggregate, and embankment fill material. It can also be used in other construction applications. RAP is a valuable high quality material that can replace more expensive aggregates and binders. The recycling of aggregates and other highway construction materials makes sound economic, environmental and engineering sense. RAP is a useful alternative to materials because it reduces the use of aggregate required in the construction of roads. The use of RAP decreases the amount of construction debris placed into landfills and does not deplete nonrenewable natural resources such as aggregate (Wright, 2006).

The practice of recycling RAP material for construction is so widely used that RAP has been called the most recycled material in the world. If this material could be re-used on site as a sub-base or base material it would reduce the environmental impact, reduce the waste stream, and reduce the material transportation costs associated with road maintenance and construction (Malpass, 2003).

The literature findings developed by Cosentino et al. (2008) about RAP applications including base, sub-base and sub-grade work showed that various processing methods have been used. Six of ten works cited used milled RAP. The top size was 1.5 inch (37.5 mm) with generally less than 2% passing sieve #200 (0.075 mm). Optimum moisture content ranged between 5.4% and 8.5%, these values are typical of granular materials. Low densities were obtained for 100% RAP, blends and special compaction procedures produced higher values.

Locander (2009) performed a comprehensive testing program to develop pavement design guidelines for using RAP in a base course. The testing program included grain size, Atterberg limits, permeability, moisture-density, asphalt content, resilient modulus and Hveem resistance values. The results indicated that RAP should have a top size of 2 inches (50 mm), a plasticity index below 6%. It was also concluded that RAP has good to excellent drainage characteristics based on the FHWA drainage guidelines.

Mokwa and Peebles (2005) evaluated the changes in engineering properties of granular soils from various sources in Montana after blending them with RAP. The research focused on engineering properties including compaction, gradation, strength, stiffness, permeability, and resistance to degradation. Milled RAP was mechanically mixed at percentages of 20%, 50% and 75% by weight with four aggregates; three of which were mechanically processed materials meeting the crushed base course specifications, and the fourth being a natural gravel material. The grain size distributions for the unblended materials showed that they were well-graded, while RAP material displayed a well-graded curve with 96% passing 1.5 inch sieve and 1% fines. For all four aggregates, the addition of RAP to the virgin materials resulted in an increase in the amount of particles passing the larger opening sieves and a decrease in the percentage of particles passing the smaller opening sieves. The smaller particles reduction was believed to be due to adhesion with the asphalt and the milling process used to produce the RAP material. The modified Proctor compaction tests on the four materials showed that adding RAP caused the compaction maximum densities decrease and the curves shift to the left.

Abbas (2013) carried out experiments to determine the optimum percentage of RAP that can be used as a substitute to virgin aggregate from the environmental, economical and strength point of view. It was found that the blending of up to 40% RAP materials with different local sub-base materials improves the RAP mechanical characteristics to meet the SCRB (2003) requirements for road sub-base and sub-grade fill materials and provides economical, environmental and sustainable road construction technique in Iraq.

The introduction of new asphalt milling machines in Iraq within the last few years had produced increasing amounts of reclaimed asphalt pavements materials. These amounts of RAP materials were accumulated due to milling of old asphalt pavements of roads without any useful usage. Agencies all around the world adopted the technique of recycling long time ago, while few research studies have been performed to characterize and examine the strength, durability and suitability of RAP materials in Iraq (Abbas, 2013).

The objectives of the present study are to make use of the RAP as a replacement material underneath a footing resting on soft clay. A trench of replaced soil will be extended to different depths to investigate the suitable dimensions of the trench.

EXPERIMENTAL WORK

Soil Used



Soil samples were collected from a depth of 0.50 m below the ground surface of a site in the north of Babylon south of Baghdad. The soil was subjected to routine laboratory tests to determine its properties. These tests include:

1. Grain size distribution (sieve analysis and hydrometer tests) according to ASTM D422 specifications.
2. Atterberg limits (liquid and plastic limits) according to ASTM D4318 specifications.

The test results showed that the soil consists of 14% sand, 44 % silt, and 42 % clay as shown in Figure

According to the Unified Soil Classification System, the soil is inorganic sandy clayey silt designated as CL. Table 1 shows the physical properties of the soil.

The RAP material was gained from a site in Al-Musaib city west of Baghdad. The material was subjected to sieve analysis to determine the grain size distribution shown in Figure 4. The specific gravity of the RAP material was measured to be 2.62 (ASTM D854)..

The Test Setup

1. Steel Container

The model tests were carried out in a test tank manufactured of steel with dimensions of 500 mm x 500 mm x 500 mm, made of steel plates 6 mm in thickness as shown in Figure 5. The container is sufficiently rigid and exhibited no lateral deformation during the preparation of the bed of soil and during the tests.

2. The Loading Frame

Figure 5 shows details of the complete set up which consists mainly of steel container, loading frame, dial gauges and accessories.

3. The Foundation Plates and Accessories

Two types of footing were used in the experiments; the first one is square of size 160 mm while the other is strip has a width $B = 48$ mm and length $L = 480$ mm.

MODEL PREPARATION AND TESTING

Preparation of the bed of soil

Prior to the preparation of the bed of soil, a relationship was obtained between the water content and the undrained shear strength of the soil as shown in Figure 6. This relation will help maintain the required shear strength for each model. The shear strength was measured using the Unconfined Compression test. In this study, the soil was prepared at a water content of 28.2 % which corresponds to an undrained shear strength of 13 kPa.

Following this stage, the bed of the soil was prepared as follows:

- (1) The natural soil was first crushed with a hammer to small sizes and then left for 24 hours for air-drying; further crushing was carried out using a crushing machine.
- (2) The air-dried soil was divided into 10 kg groups.
- (3) Each group was mixed gradually and thoroughly with sufficient amount of water corresponding approximately to the water content range of 25-30%.
- (4) After mixing with water, the soil was placed in layers inside the steel container and each layer was tamped with a special tamping hammer having a weight of 2 kg with dimensions of 50 mm x 50 mm in size. The final thickness of each layer was about 40 mm. The procedure was continued till the final thickness of the bed of soil.
- (5) After the completion of the preparation of the bed of soil, it was covered tightly with nylon sheets and left for four days as curing period.
- (6) A part of the soil was removed gently and replaced with RAP material. The RAP material was tamped with a special hammer having a weight of 1 kg. The dimensions of the replaced area varied between $W = B$ and $W = 1.6 B$ for square footing and between $W = B$ and $W = 2B$ and length = 480 mm for the strip footing. The depth of soil replacement ranged between 0.2 B to 0.6 B for square footing and $1/3 B$ to B for strip.

Model testing procedure

The model tests were carried out according to the testing program as follows:

1. A compression machine was used for applying static loads accuracy of 0.001 kN on the model footing and the settlement was recorded by linear variable digital transducer (LVDT) with accuracy of 0.001 mm.



2. This procedure was repeated for many times to get more accurate readings
 3. Loads were then applied through a loading disk with a stress controlled style in the form of load increment of 50 N. Each load increment was left for 10 minutes.
 4. The load increments were continued until the total settlement reached 24 mm in square footing and 34 mm in strip footing.
 5. During each load increment, the readings of LVDT were recorded.
- Figures 7 to 10 show the procedure of loading model footings.

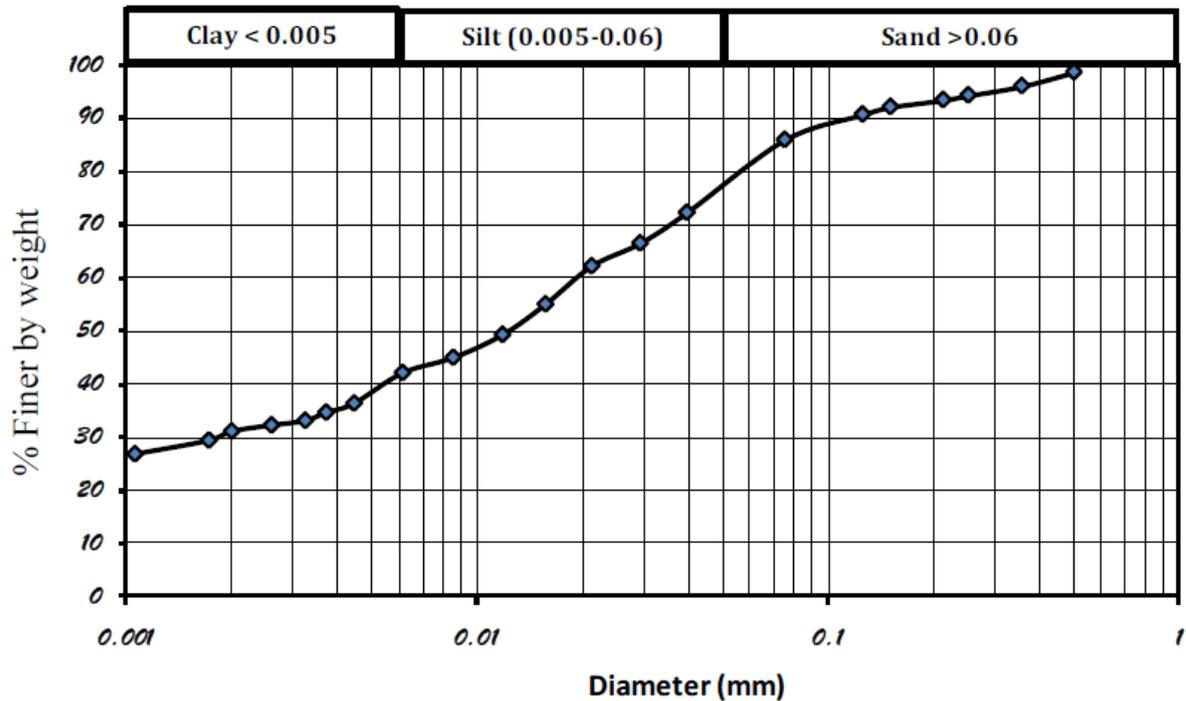


Fig. 3: Grain size distribution of the soil.

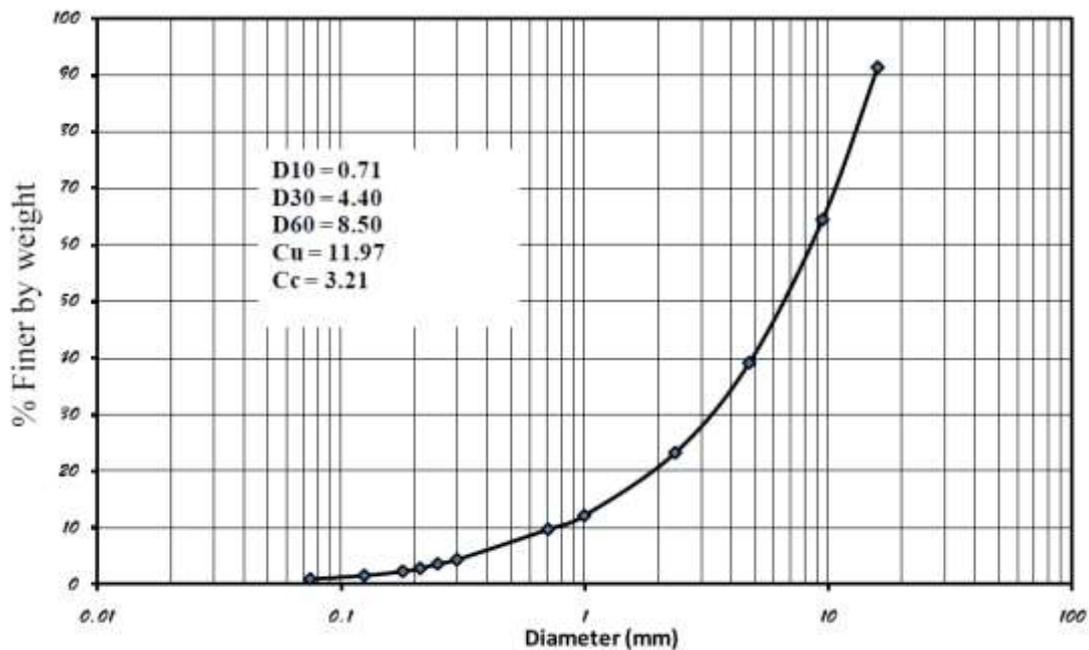


Fig. 4: Grain size distribution of the RAP material.

*Table 1: Properties of soil.*

Properties	Value
Liquid limit, L.L.(%)	35
Plastic limit, P.L. (%)	20
Plasticity index, P.I. (%)	15
Specific Gravity	2.62
Maximum dry unit weight (kN/m^3)	18.1
Degree of Saturation	100%
Optimum moisture content	10%
Sand content	14
Silt content	44
Clay content	42
Classification (Unified Soil Classification System)	CL

*Fig. 5: The steel container and the compression machine.*

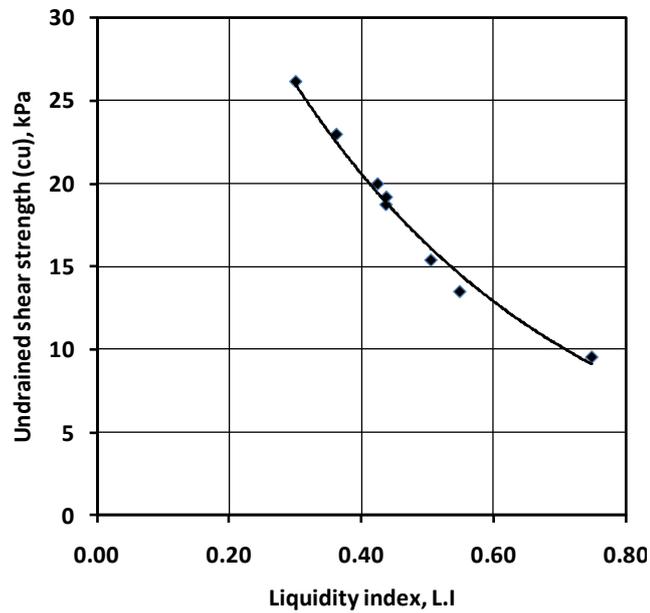


Figure 6: Shear strength-liquidity index relationship.



Fig. 7: Loading process of a footing.



Fig. 8: Square footing on a trench of RAP ($W = 1.63 B$).



Fig. 9: Strip footing on a trench of RAP (W = 2 B).



Fig. 10: RAP material under a square footing after the loading test.

RESULTS OF TESTS

Figures 11 and 12 present the pressure-settlement relations for model square and strip footings resting on trenches of RAP material of different dimensions. It was observed that the replaced zone punches through the matrix soil, punching resistance will develop within the soft soil adjacent to the vertical perimeter surface of the replaced zone. This results in increase in the bearing capacity of the footing.

In order to obtain a unified criterion for evaluation of the bearing capacity of the footings, the criterion of Terzaghi (1947) which considers the bearing capacity as the pressure corresponding to a settlement of the footing equal to 0.1 of the footing width, was adopted.

The bearing capacity ratio is then calculated as follows:

$$BCR = \frac{\text{BearingCapacity of TreatedSoil}}{\text{BearingCapacity of UntreatedSoil}} \dots\dots\dots(1)$$

Te values of BCR are summarized in Tables 2 and 3 for square and strip footings, respectively.

Figures 13 and 14 show the increase in BCR with the increase of width and depth of the trench of RAP material. An increase in ultimate bearing capacity occurs because the potential failure surface must pass either through or around the stronger replaced zone.

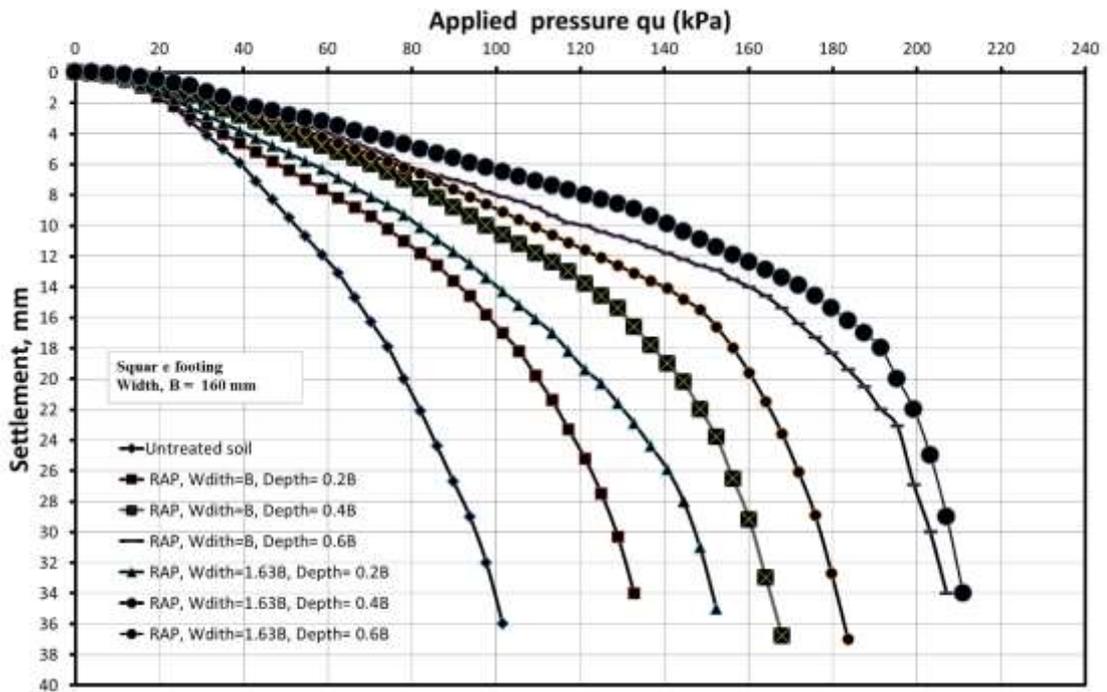


Fig. 11: Pressure-settlement relationship for a square footing $B = 160$ mm resting on a trench of RAP material.

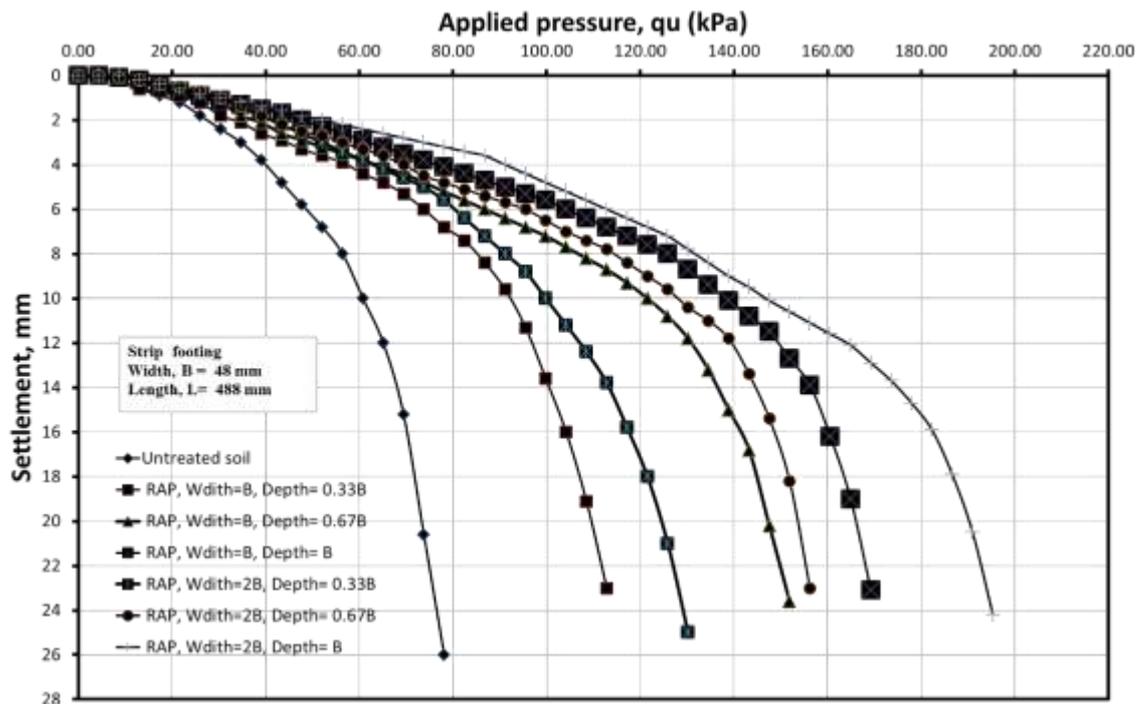


Fig. 12: Pressure-settlement relationship for a strip footing $B = 48$ mm resting on a trench of RAP material.

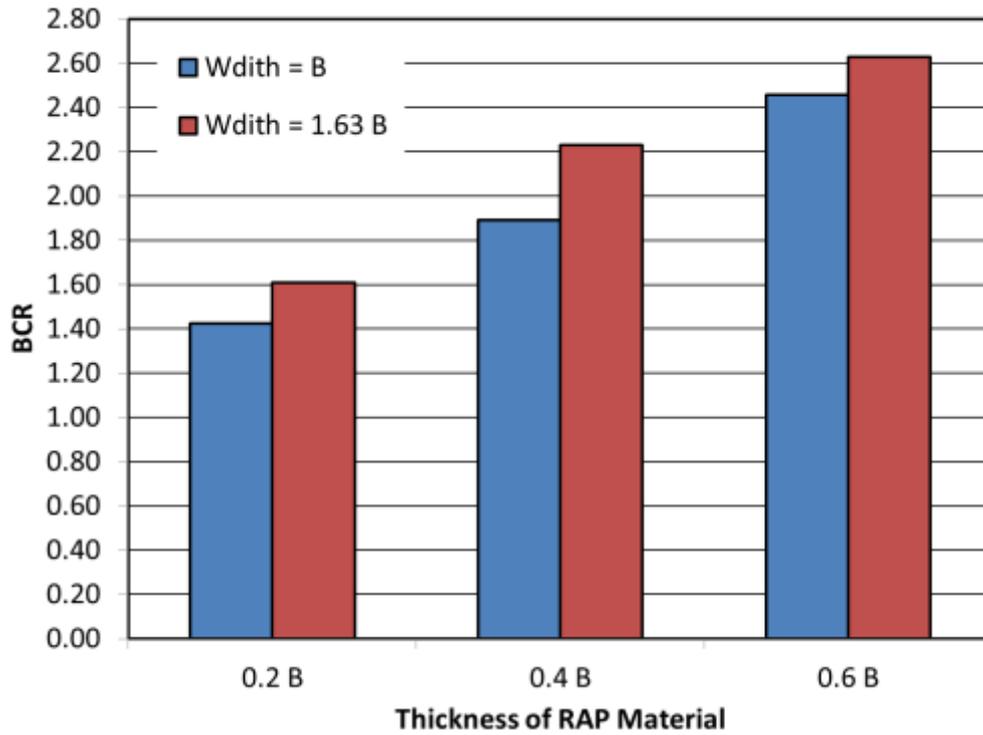


Fig. 13: Relationship between the BCR and the dimensions of the trench of RAP material. Thickness (Square footing)

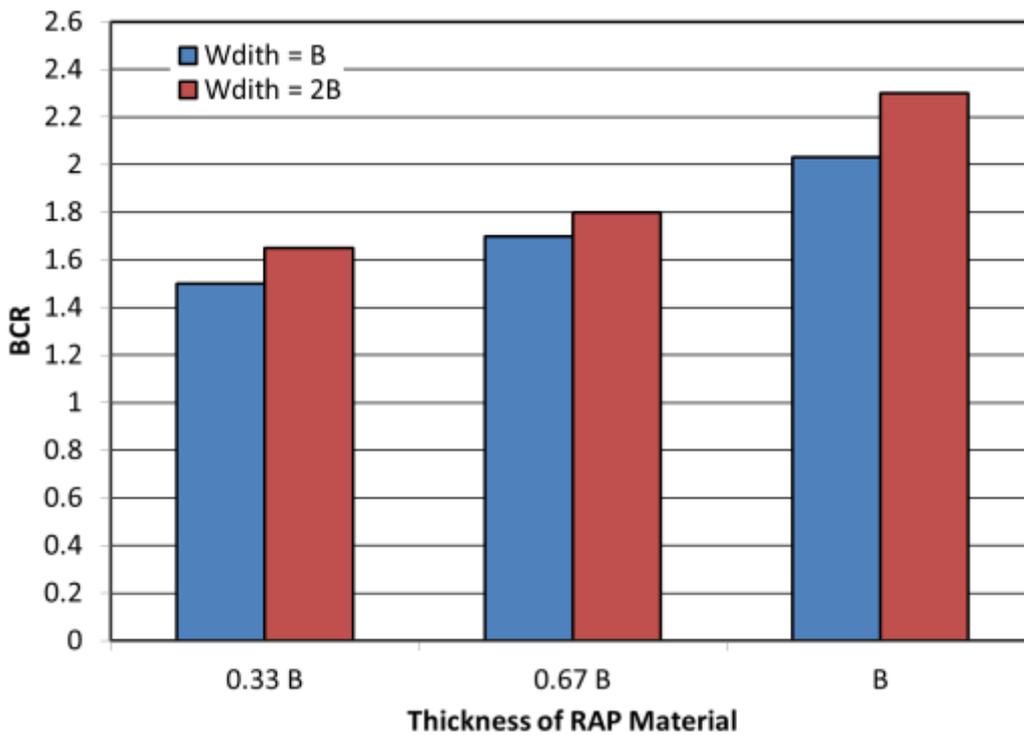


Fig. 14: Relationship between the BCR and the dimensions of the trench of RAP material. thickness (Strip footing)



Table 2: Improvement in bearing capacity of a square footing resting on a trench of RAP material.

case	Width of trench	Depth of trench	BCR
1	B	-	Untreated
2	B	0.2 B	1.42
3	B	0.4 B	1.89
4	B	0.6 B	2.46
5	1.6 B	0.2 B	1.61
6	1.6 B	0.4 B	2.23
7	1.6 B	0.6 B	2.63

Table 3: Improvement in bearing capacity of a strip footing resting on a trench of RAP material.

case	Width of trench	Depth of trench	BCR
1	B	-	Untreated
2	B	0.33 B	1.50
3	B	0.67 B	1.70
4	B	1 B	2.03
5	2 B	0.33 B	1.65
6	2 B	0.67 B	1.80
7	2 B	1 B	2.30

The bearing capacity of the square footing increased by about 42% when a trench of B width and 0.2 B depth is constructed underneath the footing. This increase becomes 61% when the trench width is made ($W = 1.63 B$). On the other hand, the bearing capacity of the strip footing increased by about 50% when a trench of B width and 0.33 B depth is constructed underneath the footing. This increase becomes 65% when the trench width is made ($W = 2 B$).

For the square footing, when the width of the trench of RAP material was limited to the footing width ($W = B$), the bearing capacity increased by 33% and 73% when the depth of trench increased from 0.2 B to 0.4 B and 0.6 B, respectively.

Confinement, and thus stiffness of the RAP material, is provided by the lateral stress within the weak soil. Upon application of vertical stress at the ground surface, the RAP material and weak soil move downward together resulting in an important concentration of stress within the RAP material. The resulting stress concentration in the RAP material is primarily due to the RAP material being stiffer than the soil.

The improvement in bearing capacity of the square footing by about 42% for a trench of B width and 0.2 B depth and 61% for trench width is made ($W = 1.63 B$) is comparable with the case of a footing resting on soft clay strengthened by stone columns. Fattah et al. (2014) investigated the behavior of the embankment models resting on soft soil reinforced with stone columns. Model tests were performed with different spacing distances between stone columns and two lengths to diameter ratios of the stone columns, in addition to different embankment heights. The embankment models constructed on soft clay treated with ordinary stone columns at spacing ratio equal 2.5 revealed maximum bearing improvement ratio equals (1.21, 1.44 and 1.7) for 200 mm, 250 mm and 300 embankment heights, respectively and maximum settlement improvement ratio equals (0.78, 0.67 and 0.56) for 200 mm, 250 mm and 300 embankment heights, respectively.

This means that making use of the reclaimed asphalt pavement (RAP) as a replacement material underneath a footing resting on soft clay is successful and comparable to stone column technique.



CONCLUSIONS

1. There is increase in the bearing capacity ratio when a trench of RAP material is constructed underneath the footing. This increase depends on the width and depth of the trench.
2. The bearing capacity of the square footing increased by about 42% when a trench of B width and 0.2 B depth is constructed underneath the footing. This increase becomes 61% when the trench width is made ($W = 1.6 B$). On the other hand, the bearing capacity of the strip footing increased by about 50% when a trench of B width and 0.33 B depth is constructed underneath the footing. This increase becomes 65% when the trench width is made ($W = 2 B$).
3. For the square footing, when the width of the trench of RAP material was limited to the footing width ($W = B$), the bearing capacity increased by 33% and 73% when the depth of trench increased from 0.2 B to 0.4 B and 0.6 B, respectively.
4. An increase in ultimate bearing capacity occurs because the potential failure surface must pass either through or around the stronger replaced zone. This eliminates the width and depth of the replaced soil zone to 2B and B, respectively.
5. For the replaced zone with RAP granular material trenches, the depth ratio has an important effect on the bearing capacity improvement ratio, which increases with the increase of depth ratio. The best practical value for the depth ratio was found to be equal to 2. Making a trench with a width (W) larger than the foundation width (B) also decreases the settlement.

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